Effects of deep excavations in soft clay on the immediate surroundings

Effets d´excavations profondes dans l´argile molle sur l´entourage immédiat

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ABSTRACT

When excavating in an urban environment, evaluation of the magnitude and distribution of ground movements is an important part of the design process, since excessive movement can damage adjacent buildings and utilities. In order to minimize the movement of the surrounding soil, a retaining wall support system is used to provide lateral support.

This article is a brief summary of the dissertation "Effects of Deep Excavations in Soft Clay on the Immediate Surroundings: Analysis of the Possibility to Predict Deformations and Reactions Against the Retaining System" presented at Chalmers in 2007, (Kullingsjö, 2007). The dissertation describes different methods for the evaluation of ground movements adjacent to a deep excavation in soft clay as well as how to estimate the lateral earth pressure that acts on the retaining system. It presents a review of:

- Soil characteristics of importance for the evaluation of deformations and earth pressure.
- Current empirical methods for estimating ground surface settlements.
- Different classical methods for calculating lateral earth pressure.
- Various soil modelling methods, with focus on the theory of elasto-plasticity.

The review is followed by an extensive case study performed at the Göta tunnel project in the centre of Gothenburg, Sweden.

Back analyses were performed in order to predict and interpret ground deformations and the development of stress changes against the retaining wall system. These analyses took the form of non-linear finite element analyses with three different constitutive models (an isotropic linear elastic Mohr-Coulomb model, the e-ADP, which is a total stress based model capable of modelling anisotropic undrained shear strength as well as non-linearity in shear, and MIT-S1, a bounding surface model based on effective stresses). The different outcomes of these three models are compared and discussed. Special focus has been placed on evaluating the parameters of the MIT-S1 model and its response compared to advanced laboratory tests.

RÉSUMÉ

Lorsqu'une excavation est effectuée dans un environnement urbain, l'évaluation de la magnitude et de la distribution des mouvements du sol est un aspect important du design, puisque des mouvements excessifs peuvent causer des dommages aux bâtiments et utilités adjacents. Dans le but de minimiser la déformation du sol, un système de support contenant un mur de soutènement est utilisé comme support latéral d´excavations profondes.

Cet article résume brièvement la dissertation "Effects of Deep Excavations in Soft Clay on the Immediate Surroundings: Analysis of the Possibility to Predict Deformations and Reactions Against the Retaining System" présentée à Chalmers 2007, (Kullingsjö, 2007). La dissertation décrit différentes méthodes d´évaluation des mouvements du sol adjacent à une excavation profonde dans l'argile molle et d´estimation de la pression latérale des terres agissant sur le mur de soutènement. Une revue est présentée concernant:

- Les charactéristiques du sol qui sont importantes pour l'évaluation des déformations et de la pression des terres.
- Les méthodes empiriques courantes utilisées pour estimer les tassements du sol.
- Différentes méthodes classiques de calcul de la pression latérale des terres.
- Différents modèles des sols avec concentration sur la théorie elasto-plastique.
- Cette revue est suivie par une étude extensive concernant le projet du tunnel Göta (Götatunnel) situé dans le centre-ville de Gothenburg, Suède.

Une analyse rétroactive a été exécutée dans le but de prédire et d´interpréter les déformations du sol et le développement des changements des contraintes contre le mur de soutènement. Ces analyses ont pris la forme d´analyses non-linéaires par éléments finis avec trois différents modèles constitutifs (un modèle isotropique linéaire élastique Mohr-Coulomb, e-ADP, un modèle basé sur les contraintes totales capable de modéliser la résistance au cisaillement anisotropique non-drainée ainsi que la non-linearité du cisaillement, et MIT-S1, un modèle avec bornes en surface basé sur les contraintes effectives. Les différents résultats de ces trois modèles sont comparés et discutés. Une attention spéciale a été portée sur l'évalution des paramètres du modèle MIT-S1 et sur l´évaluation de la réponse de ce modèle comparé à des essais avancés en laboratoire.

Keywords : Case history, Constitutive models, Deep excavation, Earth pressure, Finite element method, Ground surface settlement, *e-ADP*, *MIT-S1*

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1 BACKGROUND

When deep excavations are carried out in an urban environment, it is important to accurately predict the deformations inside and around the excavation.

Two main issues associated with such excavations are:

1) the need for the design of a temporary or permanent support system to fulfill safety demands

2) the prevention and/or minimization of damage to adjacent constructions.

A retaining system for deep excavations in the midst of existing structures is characterized by highly complex soil/structure interaction, which in turn is affected by a combination of many factors. Several authors have addressed this issue (e.g. Peck, 1969; Mana and Clough, 1981; Wong and Broms, 1989; Clough and O'Rourke, 1990; Hashash and Whittle, 1996) and some of the factors identified are listed below.

- Soil behaviour
- Excavation sequence and quality of workmanship
- The stiffness of the support system
- The geometry of the excavation (width, length and depth)
- Distance to firm stratum
- Length of embedment
- Distance to adjacent structures
- Type of foundation of adjacent structures
- The roughness between the soil and the structures
- Pore water pressure changes and accompanying consolidation in the surrounding soil

Of the above mentioned factors, soil response is the most complex. Some important characteristics are the non-linear stress strain response, anisotropy, rate effects and hysteretic behaviour. However, at present, everyday engineering work does not take all of these effects into account in the design of retaining systems for deep excavations

2 RESEARCH OBJECTIVES

The objectives of this thesis are to deepen the knowledge and understanding of how deep excavations affect the surroundings and to highlight the complex connection between stress changes and developed deformations adjacent to deep excavations. How is the stress field in the soil adjacent to an excavation affected by the work? And what will the deformation pattern look like?

A secondary objective was to increase the knowledge of how to use numerical tools such as the *FE*-method for excavation work and to explain the soil behaviour for different types of loading.

3 OVERVIEW OF THE CONTENTS OF THE DISSERTATION

A literature survey is presented and includes

- different empirical methods used for the prediction of ground surface deformation adjacent to an excavation
- crucial aspects of soil behaviour for calculating deformations and lateral earth pressure
- a review of some of the classical methods for calculating lateral earth pressure against a retaining structure
- different soil modelling methods with special focus on elasto-plasticity

An extensive field study was carried out at the Järntorget site, which is a part of the Göta tunnel project in Gothenburg. The aim of the field study was to increase knowledge, understanding and the empirical base of how deep excavation in soft clay within steel sheet pile walls affects surrounding areas. Advanced laboratory testing was carried out, which made it possible to evaluate the various soil parameters needed for the different constitutive models.

Such models have been studied in terms of their ability to describe the soil behaviour observed in advanced laboratory tests as well as that from field measurements adjacent to deep excavations. The main aim has been to evaluate the design of the constitutive *MIT-S1* model as well as its ability to describe soil behaviour under different types of loading.

The constitutive models studied were initially used in idealized excavations where the differences between the results have been elucidated. The soil models employed comprise a simple but widely used perfect linear elasto-plastic model with a failure criterion according to Mohr-Coulomb (*PLEP-MC),* the *e-ADP,* a total stress based model, and the *MIT-S1* model. After comparing the various models, the *PLEP-MC* and the *MIT-S1* were applied on the actual excavations carried out as part of the Göta tunnel project.

Constitutive relations mathematically describe how stresses and strains interact. These relations can be more or less complicated, depending on the behaviour they attempt to describe. Soil behaviour is complex and requires a sophisticated constitutive soil model. However, it is sufficient to use fairly simple models for most problems. The choice of constitutive model involves estimating the value of employing a complex model (compared to a simple one) as well as the cost of finding the input parameters to the complex model compared to the simple one.

Different groups of constitutive models are presented in Table 1. This table is incomplete, thus for more details please see the overview by Lade (2005).

Table 1: Different groups of constitutive models and their characteristics in relation to practical use

Type of constitutive relation	Example of models
Linear elastic	Hook's law
Non-linear elastic	Duncan and Chang (1970)
Elastic perfectly plastic,	Drucker and Prager (1951)
Hardening models	Druckers's Cap (Drucker *et al.*, 1957) *e-ADP* (Grimstad *et al.*, 2006)
Bounding surface models	*MIT-E3* (Whittle, 1987) *MIT-S1* (Pestana, 1994)
Multi surface models	Bubble model (Al-Tabbaa and Wood, 1989)

Easy to use and analyse. Simple description of true soil behaviour

Complex, with many input parameters. Increased possibility of describe true soil behaviour

The ability to describe true soil behaviour varies between each of these categories, and sometimes even within categories. For example, the hardening *e-ADP* model, which is based on total stresses and comprises only one yield surface and a failure criterion, is capable of describing anisotropy during undrained conditions, while the Bubble-model by Stallebrass and Taylor (1997) is not. On the other hand, the latter is capable of reproducing the small strain behaviour under undrained as well as drained conditions.

Some hardening models have one or more yield surfaces and either isotropic hardening, kinematic hardening or a mix of these. The elastic behaviour within the yield surface may be non-linear or linear. These differences will affect the outcome as well as the number of parameters (input parameters as well as internal variables) required.

Bounding surface models are based on a concept presented by e.g. Dafalias and Popov (1975). The bounding surface generates plastic strains even for reloading. Bubble models are an improvement of the bounding surface concept, as they are extended by the inclusion of an inner kinematic yield surface, which generates some plastic strain when unloaded to a specific extent.

There are models available for describing time effects such as strain rate, dependent strength and creep. These models can be based on visco-plasticity (e.g. Runesson, 1978; Zhou et al., 2005) or elasto-plasticity with an assumed time-related loading-unloading cycle for the generation of creep deformations (Vermeer and Neher, 2000). However, neither of these models is included in this work. It was assumed that the studied excavations would be completely undrained, although that is never absolutely true. Ou et al. (2000) revealed that excavations in silty clay performed over the course of a year exhibit partially drained behaviour.

3.1 *Soil behaviour adjacent to a deep excavation*

The literature survey, the extensive field study and the laboratory tests all highlighted the need to take stress history into account when analysing an excavation using the FE-method. The need for a model capable of describing the anisotropic behaviour when analysing the behaviour adjacent to a deep excavation in soft, highly plastic clay has also been demonstrated. Such soil is subject to various kinds of loading, which makes it necessary to study the shear strength in different stress paths. Figure 1 presents a schematic illustration of the stress changes in the soil adjacent to an excavation. It is obvious that there are large areas where stress rotation occurs, which makes it crucial to determine the soil behaviour associated with different kinds of loading.

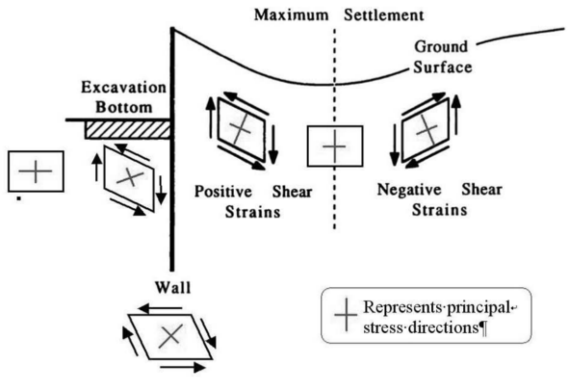

Figure 1: Schematic direction of soil shearing forces and rotation of the principal stresses, modified from Hashash, Figure 4.3-16 (1992)

When dealing with temporary excavations in soft clay, one of the most important factors is the shear strength. Under undrained conditions, which are the most common in these kinds of excavations, this is often expressed as undrained shear strength, s_u. In most soil modelling applications, this strength is treated as a constant value, independent of the different types of loading. However, most soft clays exhibit undrained strength anisotropy due to their K_0-consolidation history (Larsson, 1977; Ladd, 1991). In the laboratory, there are several different ways of evaluating s_u for different modes of shearing. An overview of these techniques can be found in Ladd (1991) and the stress path of the most common is presented in Figure 2. These tests were performed in triaxial or shear cells. When assuming isotropic shear strength behaviour related to the average shear strength, it may be impossible to reproduce an authentic stress history in combination with a realistic ratio of mobilized shear strength.

Figure 2a illustrates the stress path for the three most common undrained shear tests, CK_0UC, CK_0UE and CK_0DSS [1]. Please note that the stress path presented for direct simple shear exhibits a gradual rotation of the principal stresses as depicted in Figure 2b. In the latter, the anisotropic undrained shear strength is presented as a solid line and compared with the assumed isotropic average undrained shear strength. In this example, the initial degree of mobilized shear strength increases from 60% to 80% if the undrained shear strength from a CK_0DSS test is used as isotropic shear strength compared to if the results from a CK_0UC test. Figure 2b also reveals that the peak shear strength in the CK_0DSS test is slightly higher than that normally evaluated from this test. The maximum shear stress in the sample is the distance from the origin of coordinates and not only τ_{hor}. The direct simple shear test is usually performed without knowledge of the shear induced pore water pressure and the horizontal stress. This difference between maximum shear stress and applied horizontal shear stress is sketched in Figure 2c.

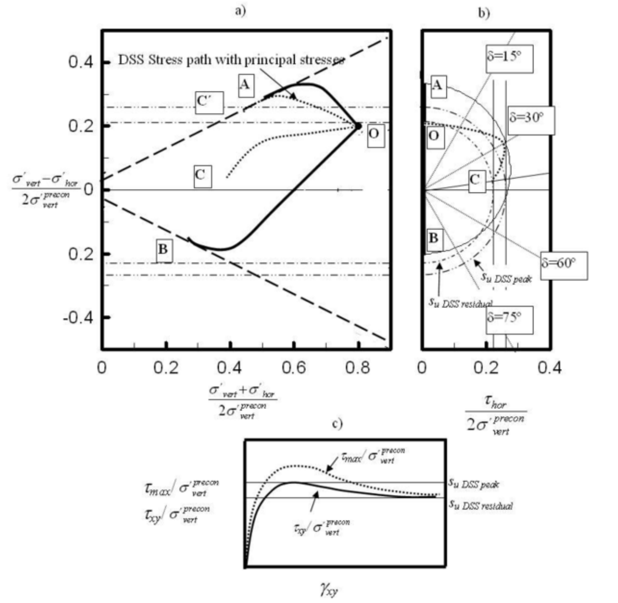

Figure 2: Idealized behaviour of a CK_0UDSS test for a clay with ϕ'=30°, $c'/\sigma'^{precon}_{vert}$ =0.03 and K_0^{nc} =0,6.
a) Stress paths: OA-CK_0UC, OB-CK_0UE, OC-CK_0UDSS (vertical and horizontal stresses) and OC´-CK_0UDSS (principal stresses)
b) Visualisation of stress rotation during a CK_0UDSS test combined with the effect of using a constant undrained shear strength value.
c) Comparison of "true" undrained shear strength with that usually evaluated from a CK_0UDSS test

Since all three stress paths occur during an excavation in clay, it is obvious that anisotropic behaviour is an important part of the overall behaviour.

4 CONSTITUTIVE MODELS

The three different constitutive models employed in the present work will be briefly described below.

4.1 *Perfect linear elasto-plastic behaviour or ideal elasto-plastic behaviour, PLEP-MC*

In this work a simple but commonly used model, in addition to more advanced models, was employed. The former is a

[1] Active shear test: CK_0UC-Consolidated for a K_0 stress situation and sheared undrained by Compression.
Passive shear test: CK_0UE- Consolidated for a K_0 stress situation and sheared undrained by Extension.
Direct simple shear test: CK_0DSS-Consolidated for a K_0 stress situation and exposed to Direct Simple Shear undrained.

perfectly linear elastoplastic model with a yield criterion based on Mohr-Coulomb's failure criterion. The plastic behaviour is controlled by a non-associated flow rule. Due to the way in which the model was used, plastic strains developed in the same manner as with an associated flow rule (Brinkgreve, 2002).

4.2 *e-ADP*

The e-ADP is a total stress based model developed by Grimstad (2005). It is briefly described here and more information can be found in the literature (Grimstad, 2005; Grimstad et al., 2006).

The reason for introducing a total stress based model is that a model capable of capturing anisotropic behaviour under undrained conditions will require simpler constitutive relations than those of a model based on effective stresses.

The formulation is limited to plane strain, *PS*, conditions and based on the work carried out by Athanasiu (1999) and Andresen and Jostad (1999). An elliptical yield surface is used, and loading from the initial state generates varying amounts of plastic strain depending on both the size and the direction of loading. The *e-ADP* model uses $s_{u\ comp}$, $\tau^{max}_{xy\ DSS}$ $\tau^{max}_{xy\ DSS}$ and $s_{u\ ext}$ as direct input parameters for strength as well as the amount of engineering shear strain at failure based on the CK_0UC-, CK_0UE- and CK_0UDSS- tests, in combination with an elastic shear modulus that defines the stress-strain response. Hardening is defined by the equation proposed by Vermeer and Borst (1984) and provides a smooth transition from initial mobilised shear stress to failure.

4.3 *MIT-S1*

The *MIT-S1* model is capable of simulating small strain stiffness, non-linear elasticity, non-linearity in shear and the development of shear induced excess pore water pressure.

The *MIT-S1* is a constitutive model presented by Pestana (1994) and capable of describing the effective stress-strain-strength behaviour of uncemented soils. It contains two separate sets of parameters for cohesive and non-cohesive soils. The model is an extension of previous work carried out at MIT (Kavvadas, 1982; Whittle, 1987) and has three basic components:

1) An elasto-plastic model with a single yield surface defined as a function of a normally consolidated soil. The plasticity is described by a non-associated flow rule, while hardening rules define the evolution of anisotropic stress-strain properties.
2) Small strain stiffness is described by a non-linear relation and the hysteretic effects caused by an unload-reload cycle are incorporated.
3) A bounding surface that controls the plasticity of over consolidated soils. The surface location is dependent on the stress path and history.

The difference between a bounding surface and a traditional elasto-plastic model is that elasto-plastic deformations occur with all loading, even within the bounding surface. Mapping rules control the magnitude of the plastic deformations within the bounding surface. The amount of plastic behaviour depends on the proximity of the current stress state to the bounding surface.

Failure conditions in *MIT-S1* are represented by an isotropic function proposed by Matsuoka and Nakai (1974). If the stresses are transformed to principal stresses, this surface can be visualized, Figure 3. The failure surface is denoted *MN* and compared with Mohr-Coulomb's failure surface, *MC*.

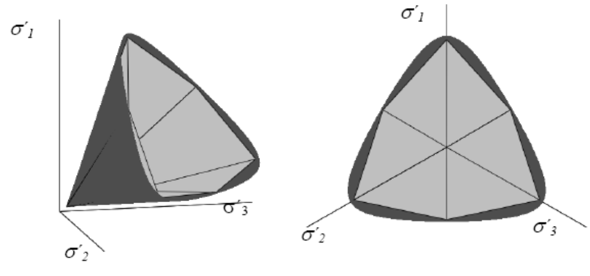

Figure 3: Visualisation of the failure surface used in the *MIT-S1* model. a) 3D plot, b) Deviatoric shear plane (Dark grey failure surface in *MIT-S1*, Light grey failure surface in accordance with Mohr-Coulomb)

The bounding surface has the shape of a distorted lemniscate. For high over consolidation ratios the surface has a similar shape to that of the *MN* failure surface, which makes it more suitable than its predecessor, *MIT-E3*, for describing the behaviour of highly over consolidated clays. In the case of smaller *OCR* values, the surface becomes more circular around the anisotropic axis, defined by the K_0^{nc} -value. In Figure 4 the bounding surface is presented in *s´-t* space and the red line represents the shape of the surface in the axi-symmetrical plane. From this perspective it is obvious that the undrained shear strength is higher when $\sigma'_z > \sigma'_x$, e.g. plane strain conditions, for a comparison see Figure 4.

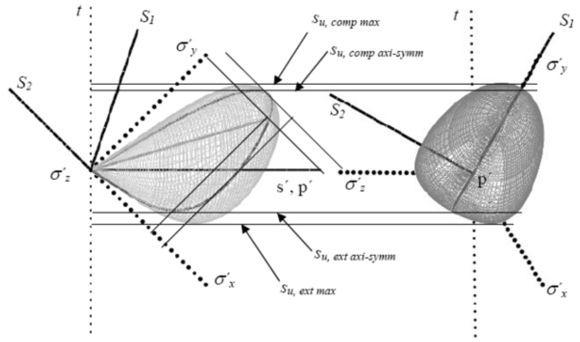

Figure 4: Undrained shear strength defined by the bounding surface. The undrained shear strength from axi-symmetric conditions as well as maximum undrained shear strength dependent on the intermediate principal stress. a) s´-t space, b) view along the s´-axis.

A detailed description of the MIT-S1 model is provided in (Pestana, 1994; Pestana and Whittle, 1999; Pestana et al., 2002). Kullingsjö (2007) presents the formulation and the model parameter evaluation procedure. The different parameters were evaluated in accordance with the procedure described by (Pestana, 1994; Pestana and Whittle, 1999; Pestana et al., 2002).

5 IDEALIZED EXCAVATION – COMPARISON BETWEEN DIFFERENT CONSTITUTIVE MODELS

In this section the calculated results from the *MIT-S1*, *PLEP-MC* and *e-ADP* models are presented.

5.1 *Soil profile*

A soil reference profile was established in order to run reference calculations for a typical excavation. The chosen soil profile consisted of a 1 m fill upon a thick clay layer with a ground water table 1 m below the ground surface. The unit weight of the fill was set to 18 kN/m^3 and the clay to 16 kN/m^3. The water pressure was assumed to be hydrostatic. The preconsolidation pressure was established by means of *OCR*=1.25, which is a typical value for the clay deposits on the Swedish west coast, and a constant value of σ'^{precon}_{vert} in the upper 4 metres of the clay layer. The *MIT-S1* constitutive model was calibrated to describe the soil behaviour observed in the Göta tunnel project. To make the comparison between the different constitutive models as independent as possible, the behaviour from the MIT-S1 model was used as true behaviour. The other constitutive models were calibrated to yield the best possible match.

The undrained strength parameters evaluated from the *MIT-S1*, with the parameter set presented in Kullingsjö (2007), were used in the numerical calculation involving the *e-ADP* model for describing the clay behaviour. The elastic shear modulus $G_{e\text{-}ADP}$ used in the *e-ADP* was set to G_{max}, in order to keep the deformations small. As a comparison, a calculation was performed using $G_{e\text{-}ADP}=2/3\ G_{max}$ to study the effect on the wall. These parameters are dependent on the *OCR* value within the soil profile. Since the *e-ADP* uses constant values for the amount of strain at failure, the clay is divided into four sub layers. The stress situation and strength are presented in Figure 5, as is the amount of strain at failure and the sub division used by the *e-ADP*.

A comparison with a *PLEP-MC* model is also presented. When small strains were expected, the G_{sec} employed in these calculations was set to one third of $G_{e\text{-}ADP}$, which accords well with the assumption when using the relation for the oedometer modulus, E_{eod}, given in Equation (1) where A=2.5.

$$E_{oed} = A \cdot 250 \cdot s_u \tag{1}$$

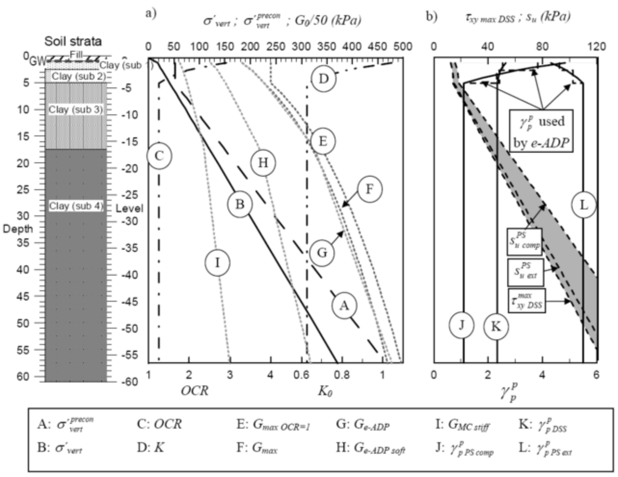

Figure 5: Stress situation and shear strength in the profile. a) Stress situation in the soil profile. b) Undrained shear strength in the profile (grey area). The amount of γ at failure for different stress paths (the solid lines represent the outcome from *MIT-S1* and the dashed lines the division used by *e-ADP*.

The interaction between the sheet pile wall, *SPW*, and the clay is modelled with interfaces, which in turn are modelled as a *PLEP-MC* material with a strength of 50% of $\tau^{max}_{xy\ DSS}$. The stiffness of the interface is set to 25% of that used when the clay is modelled as a *PLEP-MC* material.

A deep excavation with several support rows was analysed using the same constitutive models as in the previous section. The work sequence modelled was the installation of the SPW, wished in place. After installation of the retaining wall, the excavation was carried out to level –1.5. Thereafter the excavation continued *h* metres, followed by the installation of a support row at the bottom. Another excavation of h metres was then modelled, followed by the installation of a support row. This was continued to an excavation depth of 24 metres unless failure was reached. The retaining wall was set to correspond to the stiff combined wall, *HZ 975 D-12/AZ18*[2], with a length of 30 metres. The distance to firm layers was set to 60 metres. The aim was to study how the different methods of modelling clay behaviour affect that of the wall and the ground surface behind it. The fill layer was treated without any strength and merely as a load on the clay layer.

Figure 6 presents the result of an excavation to a depth of 16.5 m. Characteristics of the soil profile are provided in Figure 5, the unsupported height, h_u, is 1.5 m and the spacing, h, is 2.5 m. In Figure 6 it should be noted that Case A, in which the *MIT-S1* is used, yields smaller deformations than the other two calculations. When studying the ground surface settlements, the calculation using the *e-ADP* appears to coincide with the outcome of Case B (*PLEP-MC*) close to the wall, although further away from the excavation the settlement is approximately equal to those in Case A. The lateral deformation of the wall is approximately the same for Cases B and C, the only difference being that the deformation at the foot of the *SPW* is less in case B, due to the fact that the strength used in the *e-ADP* exceeds that employed in the *PLEP-MC* model, $s^{PS}_{u\ ext} > \tau^{max}_{xy\ DSS}$.

When studying the active earth pressure it is obvious that in Case A it developed for smaller deformations than in the other two cases and that the horizontal stress increased very quickly from developed active lateral earth pressure to a stress situation where σ_h is approximately equal to σ_v. This is an effect of using a constant value, in this case a very high value, of G_{el} and an isotropic hardening function. Very small movements against the retained side are necessary for the development of high horizontal stresses. The red lines in Figure 6 represent the limit values of the lateral earth pressure according to classical theory and the initial values of the vertical and horizontal stress. A more detailed analysis is provided in Kullingsjö (2007).

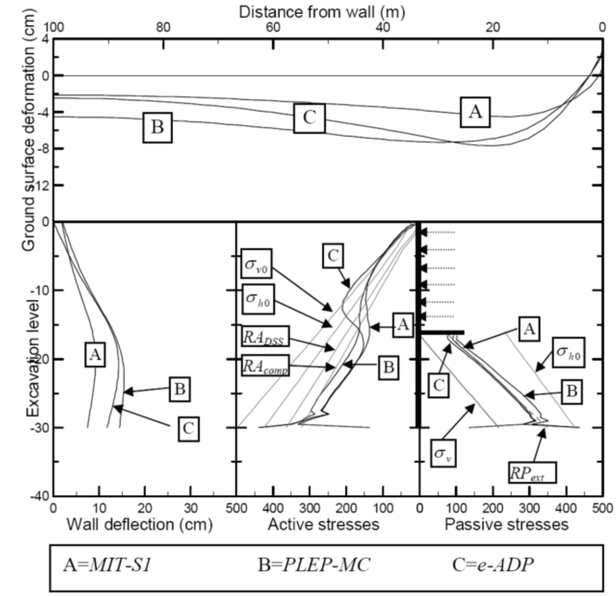

Figure 6: Comparison between three different soil models, MIT-S1, PLEP-MC and e-ADP. L=30 m, H=16.5 m, h_u=1.5 m and h=2.5 m

The strain levels obtained by means of the different calculations are also presented in Figure 7. It is obvious that the stiffness used in the *PLEP-MC* calculation is in the range of the obtained strain levels, cf. Figure 8. Obtained strain levels are in the range of 0.1 to 0.5%, which corresponds with $E_{oed} = 625 \cdot s_{u\ DSS}$. However, the strain levels within the pit are slightly higher than 1.0 % which corresponds with $E_{oed} = 250 \cdot s_{u\ DSS}$. If the strain levels obtained from the calculation using the *MIT-S1* model are deemed to be accurate, the stiffness could have been approximately twice as high on the retained side.

[2] For information about this profile please contact the manufacturer.

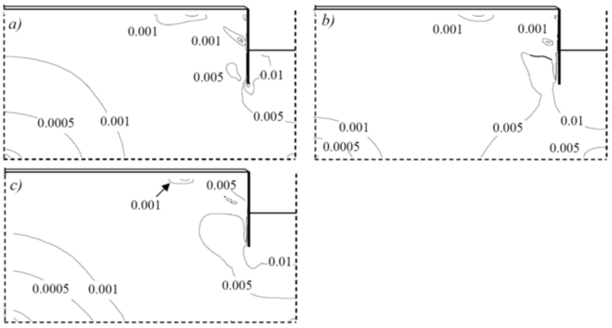

Figure 7: Strain levels, γ, for an excavation, H=16.5 m, h_u=1.5 m and h=2.5 m. a) MIT-S1. b) PLEP-MC. c) e-ADP

6 NUMERICAL ANALYSES OF DEEP EXCAVATIONS AT THE GÖTA TUNNEL

Numerical calculations are presented for section 1/470 south. These calculations are divided into 2 different categories:

Class A: Numerical simulations based on the soil properties presented by the Swedish Road Administration. Calculations of this kind can be compared with a pre-calculation used to predict the behaviour of the wall system. Numerical pre-calculations were made by the author and colleagues at Skanska prior to the design of the sheet pile walls. However, the calculations presented here contain the working sequence changes that took place during construction.

Class B: Numerical simulations, where the clay is represented by the *MIT-S1* constitutive model. The working sequence was the same as that employed in the Class A calculations.

6.1 *Class A*

In these calculations, a linear elasto-plastic constitutive model with a non-associated flow rule, referred to as the *PLEP-MC* model, represented all soil layers. The plastic behaviour was defined by the Mohr-Coulomb (*MC*) failure criteria. The calculation was made on the basis of the parameters set out in the contract between the Swedish Road Administration and the contractor, the only difference being that stiffness was increased in order to model the unloading behaviour.

The given value for the constrained modulus, E_{eod}, was a function of the undrained shear strength, s_u, Equation (2). The contractor increased the value of E_{eod} by a factor of 2.5 due to the fact that the shear strain around the excavation was expected to be small.

$$E_{oed} = 250 \cdot s_u \tag{2}$$

Empirical relations for the shear modulus of Swedish clays can be found in (Andreasson, 1979; Larsson and Mulabdic, 1991; Larsson, 1994), The relations given here are based on the work by Hardin and Drnevich (1972) and Andreasson (1979).

Hardin and Drnevich (1972) presented a modified hyperbolic relation for G_{sec} based on the G_0 and the number of load cycles, *N*. If *N* equals 1, the relation for G_{sec} will be as in Figure 8. The initial shear modulus is here based on the work presented by Andreasson (1979). Figure 8 indicates that the increase in stiffness made by the contractor may be suitable if the strain level is less than 0.5% and that the stiffness suggested by the client represents a strain level of about 1%. The use of a higher stiffness was partly a consequence of the great demands on the deformation pattern. The requirements on the movements did not allow for strains as high as 1%.

The value of s_u presented by the Swedish Road Administration is mainly based on field vane investigations and the characteristic value is reduced based on the number of tests. The s_u had a very low increase in strength per depth, thus making it obvious that the ratio between s_u and σ'^{precon}_{vert} decreases in line with depth. This holds true even when changes in *OCR* are taken into account. Please note that the ratio have been found quite constant from the laboratory testing within this work, however this was not used in the Class A prediction.

As a consequence of the given s_u the initial stress in the calculation had to be modified in order to avoid the generation of plastic behaviour prior to the start of the excavation. When using a *PLEP-MC* model with failure criteria based on a constant value of s_u, it is of major importance to employ relevant combinations of initial stresses and shear strengths.

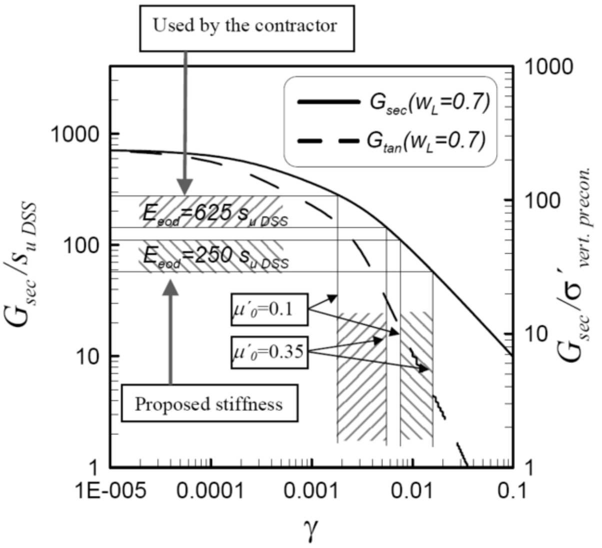

Figure 8: Empirical relations for highly plastic Swedish clays in terms of shear modulus compared with strain independent shear modulus.

6.2 *Class B calculations*

These calculations were performed by using the *MIT-S1* to describe the clay behaviour. The properties were evaluated by Kullingsjö (2007). The working sequence was the same as that in the Class A calculation.

6.3 *Results from the Class A and Class B calculation*

The results of the different calculations are presented below, in addition to the monitored lateral earth pressure, Figure 9, and the monitored deformation, Figure 10.

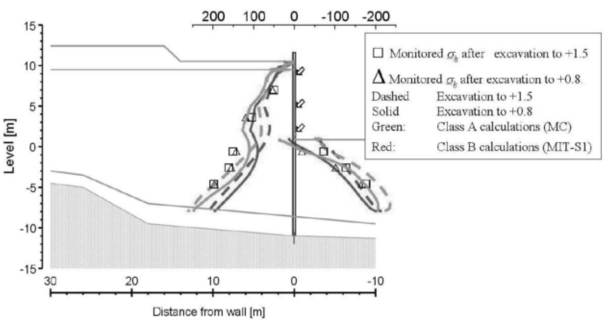

Figure 9: Comparison between monitored and calculated σ_h. Green curves: Class A calculations. Red curves: Class B calculations

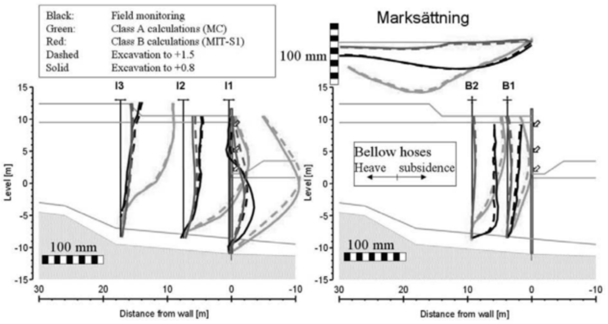

Figure 10: Comparison of calculated and monitored deformations caused by the first stages of the excavation. a) Vertical deformations. b) Horizontal deformations. Green curves: Class A predictions. Red curves: Class B predictions

The results of the Class A and B calculations are presented in Figure 9 and Figure 10. Black curves represent the monitored deformations/pressures, green curves the results of Class A and red curves Class B calculations.

The calculated earth pressure was found to have excellent agreement with the monitored earth pressure, especially on the passive side. In fact, the simpler calculation has a slightly better agreement compared to the monitored ones. However, when a single point in the soil mass is studied in terms of earth and pore water pressure, it becomes obvious that the class B calculation provides more realistic results.

From the comparison between the results of class A and B calculations, it is evident that the former greatly overestimated the monitored deformation. It is important to bear in mind that the monitored vertical deformation is to a great extent caused by volume loss in the underlying sand layer due to the installation of anchors. Neither calculation A nor B models this phenomenon.

7 CONCLUSIONS

Back analyses were performed in order to predict and interpret ground deformations and the development of stress changes against the retaining wall system. These analyses took the form of non-linear finite element analyses with three different constitutive models (an isotropic linear elastic Mohr-Coulomb model, *e-ADP*, a total stress based model capable of modelling anisotropic undrained shear strength as well as non-linearity in shear, and *MIT-S1*, a bounding surface model based on effective stresses.

The outcome of the analyses shows the advantage, compared to simpler models, of using finite element methods in combination with an advanced soil model, such as the *MIT-S1*, capable of simulating small strain stiffness, non-linear elasticity, non-linearity in shear and the development of shear induced excess pore water pressure. The analyses also show the importance of combining *FE*-analyses with empirical methods for estimating ground surface settlements.

Detailed *FE*-modelling is, however, not sufficient. Close collaboration between the geotechnical consultant and the contractor is necessary in order to ensure a reliable construction that behaves in an acceptable manner. Monitoring, continuous follow up and comparison between monitored and calculated behaviour are essential and offer the possibility to make adjustments in the design if necessary. Other engineering activities adjacent to the wall, e.g. piling, ground water lowering and effects from anchor installation, have to be taken into account when making predictions and when analysing the behaviour of an excavation. Back analyses were performed in order to predict and interpret ground deformations and the development of stress changes against the retaining wall system. These took the form of non-linear finite element analyses with three different constitutive models (an isotropic linear elastic Mohr-Coulomb model, the *e-ADP,* which is a total stress based model capable of modelling anisotropic undrained shear strength as well as non-linearity in shear, and *MIT-S1*, a bounding surface model based on effective stresses).

The outcome of the analysis demonstrates that finite element methods in combination with an advanced soil model such as the *MIT-SI*, which is capable of simulating small strain stiffness, non-linear elasticity, non-linearity in shear and the development of shear induced excess pore water pressure, are advantageous when compared to simpler models. The analyses also highlight the importance of combining *FE*-analyses with empirical methods when estimating ground surface settlements.

Detailed *FE*-modelling is, however, not sufficient. Close collaboration between the geotechnical consultant and the contractor is necessary in order to ensure a reliable construction that behaves in an acceptable manner. Monitoring, continuous follow up and comparison between monitored and calculated behaviour are essential and offer the possibility to make adjustments in the design if necessary. Other engineering activities adjacent to the wall, e.g. piling, ground water lowering and effects from anchor installation, have to be taken into account when making predictions and when analysing the behaviour of an excavation.

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[4] NTNU – The Norwegian University of Science and Technology

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